UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING APPEAL BOARD

In the Matter of	)	
	)	
LOUISIANA POWER AND LIGHT COMPANY	)	Docket No. 50-382
	)	
(Waterford Steam Electric Station, Unit 3)	)	

AFFIDAVIT OF JOHN S. MA

I, John S. Ma, being duly sworn, state as follows:

1. I am employed as a structural engineer in the Structural and Geotechnical Engineering Branch of the Office of Nuclear Reactor Regulation, U.S. Nuclear Regulatory Commission.

2. I previously submitted an affidavit in this proceeding on November 28, 1983 which provided the evaluation and conclusions of the NRC Staff concerning the adequacy of the Waterford Unit 3 foundation base mat in light of the discovery of concrete cracking and water seepage in the base mat. A statement of my professional qualifications is attached to my November 28, 1983 affidavit.

3. This affidavit is being provided to the Appeal Board pursuant to its request in ALAB-786 (October 2, 1984) seeking an updating of my views with respect to the cracking of the Waterford Unit 3 base mat. See ALAB-786, slip op. at 14 n. 13.

4. My views with respect to the Waterford 3 base mat cracking issue are contained in the attached report dated December 10, 1984 which I prepared in response to the Appeal Board's request. I hereby certify that the statements contained in the attached report are true and correct to the best of my knowledge and belief.

John S. Ma

John S. Ma

Subscribed and sworn to before me
this 12th day of December, 1984

Notary Public

My Commission Expires: 7/1/86

MAXINE H. LAIEFSKY
NOTARY PUBLIC
MONTGOMERY COUNTY, MD.

WATERFORD UNIT 3 BASE MAT

JOHN S. MA
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DIVISION OF ENGINEERING, NRR, NRC

DECEMBER 10, 1984

I. INTRODUCTION

The objective of this report is to assess the adequacy of the Waterford Unit 3 concrete base mat in light of the discovery of concrete cracking and water seepage in the mat. The nature of my involvement as well as events which occurred are described.

The behavior of the mat under dead weights (loads) is described. The results of nondestructive testing are outlined. The behavior of the cracked mat during earthquakes is qualitatively described in terms of a mathematical model and its complications are presented.

The report by Brookhaven National Laboratory (BNL) and its addendum were reviewed. Both the cause and effect of concrete cracking assessed by BNL were found to be inadequate.

Concrete cracking under and near the reactor within the Reactor Containment Building (RCB) is extensive, although noncontinuous, both in length across the width of the mat and in depth, but its widths are unknown. Some of the cracks outside of the RCB, which were believed to be jointed by six cracks within the RCB, were found to be extensive, and their crack widths were estimated to be less than 0.007 inches. The potential problems of reinforcing steel corrosion and durability of concrete affected by concrete cracking are described. The permissible

crack widths listed in codes and specifications for reinforced concrete structures are presented for comparisons with the existing crack widths in the mat.

Information has not advanced to a sufficient degree to conclude that the cracked mat is adequate and will be adequate in terms of its functionality. However, it can be concluded that the mat will be adequate if successfully repaired and with the implementation of a monitoring program to assure the continued successful performance of the repaired mat. No further analysis will be required except perhaps in connection with a particular selected repair method.

II. HISTORICAL EVENTS

II.1 Involvement

Without any prior involvement in or knowledge of the Waterford Unit 3 project, I was assigned in about October 1983 to resolve the safety issue of the concrete mat cracking and water seepage. The assignment was to complete the review with a lead responsibility in about one and one-half months. I completed my review in time and submitted my affidavit on November 28, 1983.

After completion of that affidavit, I was not officially assigned to work on the Waterford Unit 3 project until March 1984. In between those times, I was given a copy of Mr. Phearson's memo and a copy of a Gambit newspaper article, both alleging construction deficiencies. I was assigned to report to a section leader, and to a branch chief who led the project. It was made clear to me that I would not be the one to lead the project or to write an affidavit for it, as I had done the

previous year. Consequently, my involvement on the project was intermittent due to the nature of the assignment. I attended some of the meetings with the applicant, design engineers, and NRC consultants when requested by my superiors. I provided opinions in either oral or written form in accordance with the requests.

On October 23, 1984, I was given a copy of the Appeal Board's Memorandum and Order (ALAB-786). On October 24, 1984, I was requested to submit a draft affidavit by October 30, 1984. I submitted my draft affidavit on October 30, 1984. On November 9, 1984, I was given a copy of NRC Staff's Answer to Joint Intervenors' Amended and Supplemental Motion to Reopen Contention 22, dated August 7, 1984, and a copy of the BNL report, dated July 18, 1984, and was requested to read them and present a full statement of my opinions as to the adequacy of the basemat. This report was written in accordance with that request.

II.2 Summary Of My Earlier Affidavit

In my earlier affidavit (Reference 1), I assessed and documented the adequacy of the Waterford Unit 3 foundation base mat in light of the discovery of concrete cracking and water seepage in the mat. In that affidavit, I concluded that the cracks and water seepage did not represent a challenge to the structural integrity of the foundation base mat and recommended that a surveillance program for corrosion of reinforcing steel bars be initiated to provide additional assurance to the continued structural integrity of the mat. Before reaching that conclusion, I reviewed two reports by Harstead Engineering Associates (HEA), Mr. John Collins' letter, and the final draft of Dr. Raman Pichumani's affidavit; this is mentioned in my earlier affidavit.

The cracks as reported by HEA were small in width (between 2 to 5 mils; HEA Report No. 1, p. 8). The depth of these cracks should be shallow because the approximate (tensile) stress in the top rebar, 1200 psi as reported in HEA Report No. 1, p. 24, would indicate that the concrete was uncracked at the level surrounding the rebar, which is located approximately 4.25 inches below the top surface of the mat. The HEA reports also indicate that the mat possesses ample margins of safety against bending (flexure) failures and adequate margins of safety against shear failures (HEA Report No. 2, pp. 18 and 19). Furthermore, the HEA reported that a repair program had been established to deal with construction deficiencies and the repair program was believed to be effective (HEA Report No. 1, pp. 6 and 7). A refined analysis to incorporate concrete cracking had been considered and was later assessed by HEA to be unnecessary because the amount of cracking was insignificant in affecting structural <u>response</u> and <u>integrity</u> in comparison with the original assumption of homogeneous uncracked concrete used in analyses (HEA Report No. 2, pp. 17 and 21). Based on the information available to me at that time including the understanding gained during my audit at EBASCO Services, I found no evidence that would render the HEA conclusions to be inaccurate or unreasonable.

The field construction was adequate and acceptable because it was performed in accordance with approved procedures, specifications and regulations, as stated in a letter by Mr. John Collins, Regional Administrator of NRC Region IV. The letter was attached to my earlier affidavit.

The foundation underlying the mat was evaluated as adequate to support the mat and the structures and equipment on top of the mat (Pichumani's affidavit, Q4 and A4). Pichumani's affidavit (Q6 and A6) also stated

that the applicant had given adequate consideration in its analysis of the mat to accommodate the variations in soil stiffness.

Considering the positive reports from Collins on field construction, Pichumani on foundation soils, the NRC Senior Resident Inspector on "hairline" cracks (Ma's affidavit, Q4 and A4), and my reviews (Ma's affidavit, Q7 and A7), I concluded in my affidavit, dated Novmeber 28, 1983, that the mat was still adequate to perform its function.

II.3 Site Visit

Dr. John Chen, a geotechnical engineer, my section leader, and my branch chief and I visited the Waterford Unit 3 site on March 27, 1984. This was my first visit. I observed cracks on the ring wall and wet cooling tower walls. These cracks had not been mapped or brought to my attention until that time. Some of the cracks on the walls were jointed by a crack on the mat. I was not sure those cracks on the wall were caused by a relative displacement of the mat across the crack faces on the mat; I was certain that a relative displacement across the faces of the crack on the mat could propagate the existing cracks upward on the walls.

At the site, I reviewed construction records and interviewed various individuals who participated in the construction of the foundation base mat. Based on the review of construction records and interviews, I found that despite the effort of the applicant's quality assurance organization, the first three blocks of concrete placement, where major cracks occurred, did have quality control problems. These problems included (1) dropping concrete beyond 5 feet height at times, (2) using a concrete vibrator improperly and providing insufficient vibration, and (3) one instance of sledge hammering a reinforcing bar to make room for a concrete-placing elephant trunk, thus transmitting shock waves to the

concrete below through vertical reinforcing bars. Such actions could lead to cracking concrete or creating voids around reinforcing bars. Deficiency modifications were written for observed cracking and honeycombing detected in vertical walls of the concrete blocks on concrete surface, and the records showed that the deficiencies were repaired. Action to eliminate such deficiencies finally resulted in a stop work order issued by LP&L after the concrete placement of the first three blocks. When the construction was resumed, quality control was reported to be improved. The construction problems were later handled by Mr. L. Shao's team.

II.4 Preliminary Evaluation and Proposed Action

A few days after we returned from the site, Dr. Chen reported in a meeting that the soils underneath the mat were nonuniform and had experienced differential settlements due to dead leads and uneven ground movements because of groundwater level changes. He also reported that the groundwater levels had changed significantly three times during construction, ranging from 20 to 30 feet. Dr. Chen's statements were later documented (Reference 2).

The structural effect on the mat due to nonuniform soils can be illustrated as follows. The portion of the mat resting on top of weaker soils, such as Strip No. 2 in Chen's report, tended to settle down more than the other portion of the mat resting on top of stronger soils, such as Strip No. 1. If there were a vertical gap in the mat of its entire length and depth, no stresses would be induced as a result of these uneven settlements, because the two pieces of the mat could not hold or restrain each other and would be free to move. Since the mat was actually a monolithic piece, the continuity would restrain one portion

of the mat to settle down more than the other at their interface; this restraint introduced stress in the mat when uneven settlements occurred.

Stress was also introduced in the mat due to uneven soil pressure which was associated with hydrostatic pressure change in conjunction with the nonuniform soils under the mat. The phenomenon here is that soils with uneven magnitude of force pushed up against the mat instead of the mat settling down as previously described.

Stresses associated with nonuniform soils during construction were not calculated in the original analyses and thus were not designed for. It is appropriate to note that condition (2) of A6 in Pichumani's affidavit represents a future state of soil conditions, not the state during construction. Neither the magnitude nor the distribution pattern of these stresses was known. The dilemma was whether or not the mat could accommodate these additional stresses during construction. The dilemma was intensified because the effects of the construction problems discovered during the site visit had not yet been evaluated. The construction problems might have seriously affected the quality of construction and might have reduced the strength of the mat.

It was difficult to assess the safety of the mat, the undefined magnitude of stress acting on it, and the doubtful strength of the mat. In the meeting after the site visit, Dr. Chen and I jointly proposed to obtain the magnitude of these additional stresses through analyses by using actual soil conditions during construction. In addition, I proposed that a direct physical examination of the mat be performed by non destructive testing (NDT). The logic was that adequacy of the mat would be proved if the following two statements were true: (1) the additional stresses associated with nonuniform soils were calculated and added to other applicable stresses and the sum of these stresses was within the

specified allowables, and (2) the NDT results did not indicate any serious internal cracks that had weakened the shear strength of the mat to an extent which might lead to a shear failure during a safe shutdown earthquake. Bending failure was less likely due to the ample factor of safety designed into the mat, and therefore was not a concern. The analyses were not performed, but the NDT was performed later.

III. ANTICIPATED BEHAVIOR OF THE MAT UNDER STATIC AND DYNAMIC LOADS

III.1 Introduction

Deflections are visible indications of the response of a structure to a load, and they are used to describe the behavior of the mat under dead load and to examine the design adequacy. This section discusses the importance of construction sequences in affecting concrete crack patterns and requiring rebar placement patterns. The importance of the interaction of the type (concentrate or uniform) of load, mat stiffness, and soil foundation stiffness is discussed, and the NDT results are outlined. The behavioral model of the cracked mat subject to earthquakes and its complication in performing dynamic analysis are also discussed.

III.2 Construction Sequence Effect

Figure 1a is a plan view of the mat supported by soils (soils are not shown). The mat is loaded with the weight of, and is restrained by, the side walls, ring wall, and structures within the ring wall including the reactor. A north-south (N-S) strip and an east-west (E-W) strip of the mat across the center of the reactor is shown in Figure 1a for later illustration purposes.

Assume that the ring wall and its internal structures including the reactor are first built on the mat. One can then visualize that the center portion of the mat will deflect downward and the edges upward with respect to the original assumed flat position. The deflected shape of the two perpendicular strips from Figure 1a is shown in Figures 1b1 and 1c1. These strips will deflect greatest among their parallel strips.

The as built mat has a thickness of 12 feet instead of a line, as shown in Figures 1b1 and 1c1. In order for the mat to deflect and bend in such a dish shape from its original flat position, the bottom fibers have to be elongated and the top fibers compressed at all cross sections except where the edges actually rise off the soils. The dish or concave type of bending is designated as positive bending, and the reverse or convex type of bending is designated as negative bending. (This sign convention is used by EBASCO, but is opposite to the one used by BNL.)

If the mat is uncracked, at the approximate center of each cross section there must be a fiber (neither elongated nor compressed) as a transition between elongated fibers on one side and compressed fibers on the opposite side during bending. This fiber is called the neutral axis. The deflected line shown in Figures 1b1 and 1c1 is considered the line connected by all the neutral axes along the strip. Since concrete is strong in compression resistance but weak in tension, rebars are usually placed in the tension side to provide or supplement the necessary tension.

As the weight of the side walls is placed, the edges of the mat deflect downward and bend the outer portion of the strips into a convex shape, as shown in Figures 1b2 and 1c2. These deflected shapes suggest that

the mat has some negative bending in the outer portion and large positive bending in the midspan. The adjacent strips in parallel to the strips shown in Figures 1a will also take the same deflected shape as shown in Figures 1b2 and 1c2, but with a smaller amount of deflection proportionally along the length of the strip. This kind of construction sequencing requires rebars to be placed in both N-S and E-W directions at the bottom of the mat, with a greater amount of rebars in the middle portion and a lesser amount at the edges to satisfy the construction stage that would produce the deflected shape, as shown in Figures 1b1 and 1c1. The concave shape of the mat may result in concrete cracking at the bottom and most likely near the center portion of the mat where the bending is the greatest. In order to satisfy the construction stage that would produce the deflected shape, as shown in Figures 1b2 and 1c2, rebars are needed at the top in the outer portion of the mat where the negative bending is indicated. Theoretically, there is no need for rebars to be placed at the top middle portion of the mat for bending where negative bending has never been generated because of the type of construction sequence. The placement of rebars at the top was for other considerations, such as concrete shrinkage phenomenon and thermal effect.

If the construction sequence is to build the side walls first, the mat will deflect into an inverted dish type with soils pushing upward. The deflected shape of the strips in Figure 1a will take the shape, as shown in Figures 1b1' and 1c1'. These two perpendicular strips will deflect more than any parallel strips. The amount of deflection of parallel strips decreases gradually in proportion to the distance to these strips, until alongside the side walls it approaches zero. This construction sequence requires rebars to be placed at the top of the mat in both N-S and E-W directions, with more rebars in the middle than at the edges. This type of deflected shape may produce tension cracks on

the top surface of the mat, especially near the center portion where the bending is the greatest. As the placement of the ring wall, reactor and other internal structures is completed, the deflected shape will be approximately the same as the previous construction sequence produced, as shown in Figures 1b2 and 1c2.

It has been shown that although the final deflected shape of a mat may be the same due to different construction sequences, the concrete crack pattern and rebar placement requirement are influenced by the construction sequences. Different load paths may require different rebar placement and may produce different crack patterns in concrete structures.

When the backfill soil level reaches about 6 feet above the top surface of the mat, additional backfill soils produce positive bending near the edges of the mat. This phenomenon can be seen by comparing the N-S bending (moment) distribution along an E-W strip across the center of the reactor, as shown in Figures 2 and 3. The main difference between the two figures is that no backfill soil was assumed in calculating the bending moment in Figures 2, while a full backfit soil was assumed in Figure 3. The negative bending moments, although small in Figure 2 have been reversed into positive bending moments in Figure 3.

During my audit at EBASCO in 1983, it was my understanding that the actual construction sequences were planned and executed so that stresses in the mat and soils were minimized, and the backfill soils were placed gradually as the construction proceeded.

III.3 Adequacy of Mat Design As Checked by Deflected Shape

A concrete mat supported by soils requires a three-dimensional approach for structural analysis, and employs advanced mathematics to solve partial differential equations. Because of this mathematical complication, engineers usually rely on computer programs developed by mathematicians to obtain solutions, such as the magnitude of bending moments. Whether or not the computer results are logical needs to be critically examined before the results are used for sizing cross sections and proportioning rebars. Errors in, and misuse of, computer programs have been reported, but I am not aware of any such problems concerning Waterford Unit 3.

EBASCO's computer results on the N-S bending across an E-W strip through the center of the reactor are plotted in Figure 2 with no backfill soils and Figure 3 with full backfill soils. Figure 2 shows a maximum positive bending moment of about 200 ft-k per foot width, and 800 ft-k/ft within the ring wall for a constant soil modulus and variable soil modulus, respectively, and a maximum negative bending moment of about 100 ft-k/ft and 200 ft-k/ft between the ring wall and side walls for a constant soil modulus and variable soil modulus, respectively. Figure 3 shows about the same magnitudes of positive bending moments within the ring wall, but also shows positive bending moments between the ring wall and side walls with a maximum of about 400 ft-k/ft. The reversal of this bending moment was mainly due to the difference between no backfill soils and full backfill soils assumed in the computer analysis. According to the actual construction sequence, the N-S bending moments between the ring wall and the side walls should vary along the zero line with a small variation for the assumed soil conditions.

Whether or not these bending moment patterns make sense is examined now. The two perpendicular strips in Figure 1a must deflect the same total amount at their common point. In order to have a common deflection at

the centers of both strips, the shorter E-W strip has to bend more than the longer N-S strip; hence, bending moments in the E-W direction are greater than the N-S direction. Accordingly, there should be more rebars per foot width in the E-W direction than in the N-S direction. Rebar placement in the mat is shown in Figure 4. Regarding top rebars for negative bending moments, Figure 4 shows that there is one layer of #11 bar at 6 inches and another layer of #11 bar at 12 inches, with a total steel area of 4.68 square inches per foot width in the E-W direction; there is only one layer of #11 bar at 6 inches with a steel area of 3.12 square inches per foot width in the N-S direction. Concerning bottom rebars for positive bending moments, Figure 4 shows two layers of #18 at 8 inches with a total steel area of 12 square inches per foot width in the E-W direction, and two layers of #11 at 6 inches and one layer of #18 at 9 inches with a total steel area of 11.57 square inches per foot width in the N-S direction. This is consistent with the requirements of deflected shape. Figure 4 also shows the steel area decrease in both directions in those strips away from the center strips (excluding local modifications due to additional loads). This is consistent with the phenomenon of the expected deflected shape described previously.

These comparisons provide certain confidence as to the correctness of the computer programs used by EBASCO and to the ability of EBASCO engineers who adequately modeled the real structure for mathematical analysis. Furthermore, I personally calculated both the positive and negative bending moments with loads borrowed from EBASCO to be used for design within and near the area of the ring wall and found that EBASCO bending moments were adequate for design. The EBASCO calculation for bending moment capacities was also adequate based on my spot checks. If the loads are adequately calculated, I have no doubt that the original mat design is adequate for the current soil condition. This adequacy is

limited only with respect to design where uncracked sections are assumed and when a small amount of shallow cracks is anticipated.

III.4 Interaction Among The Type Of Load, Mat Stiffness, And Soil Stiffness

The purpose of the mat is to receive concentrated loads on top surfaces and to redistribute them into a somewhat uniform load at the bottom surface before the loads are transmitted to soils. The stiffer the mat, the more uniform type of forces for the soils; the stiffer the soils, the less deflection of the mat. A mat supported by very stiff soils and subjected to uniform loads on top would not bend (or would bend very little) and would thus produce little bending moments. However, the real load on top of the mat is not uniform, and neither the mat nor the soils is very stiff.

The mathematical expression for the interaction of loading types, mat stiffness, and soil stiffness is complex. This complexity is intensified due to a changing stiffness in both concrete and soil. Concrete gains strength and stiffness with time, especially in the early stage after placement, and loses both when cracked; soil stiffness is increased with time due to consolidation. Therefore, the interaction has been existing and stress in the mat changing since the first placement of concrete.

Figure 2 illustrates changes in the magnitude of positive bending moments as a result of a change in soils stiffness alone. A change in soils stiffness from 150 pounds per cubic inch to 70 pounds per cubic inch within the ring wall resulted in a positive bending moment increase from about 200 ft-k/ft to 800 ft-k/ft. Since there is no analysis performed incorporating the actual soil condition, the maximum is not known for the stress level that the mat actually experienced during

construction. This makes later assessment on the cause of cracking more difficult.

III.5 NDT Result

A detailed NDT result is contained in a report entitled, "Nondestructive Test Evaluation of Base Mat Concrete, Waterford No. 3," by Muenow and Associates, Inc., dated October 1984. The important aspects are summarized as follows:

a. All the cracks are nearly vertical with a slight slope toward the north.

b. The depths of cracking vary from shallow to a through-crack.

c. A total of 37 surface cracks outside of the RCB were investigated and crack width estimation was performed on 7 of the 37 cracks. Crack widths in the mat were reported as less than 0.007 inches. The surface cracks were measured as between 0.003 and 0.005 inches.

d. The NDT indicated 7 internal cracks under the RCB. Six of the 7 cracks were joined by the cracks outside of the RCB and the remaining one stopped inside the RCB. The crack length along the width of the mat is quite extensive but noncontinuous. The depths of these cracks are quite deep with many through-cracks. No crack width estimation was performed on these cracks.

e. Concrete consolidation is adequate, with high compressive strength.

f. No significant seepage of water from the cracks was found during the testing period.

III.6 Behavior Of The Cracked Mat Under Dynamic Load

The response (displacement, rotation, strain, and stress) of seismic Category I structures, safety class equipment and piping, which are supported by the mat, was calculated by assuming the mat being one monolithic piece, as shown in Figure 5. The fundamental assumption in this mathematical model is that the mat will move horizontally or vertically and rotate about an axis in one monolithic piece during earthquakes. This assumption is valid if a small amount of shallow cracks are present in the mat, as envisioned in the HEA reports. The extensive cracking in both length and depth, as indicated by NDT, has raised a question on the validity of this assumption and a need for further examination.

At each cross section across the width of the mat, concrete alone provides a tensile force (strength) of about 166 million pounds, while the top rebars provide a tensile force of only 50 million pounds at their yield strength. No tensile force can be transmitted through a crack face. Without this 166 million pounds of tensile force in reserve, the reactor and its support that bridges the cracks may be called upon to transmit forces. This is not a desirable load path and should be avoided, or at least its capability needs to be verified.

A mathematical model of the cracked mat is shown in Figure 6 without superstructures for clarity. The model represents a crack with extensive length and depth in a cross section across the width or an equivalent crack representative to the sum of all cracks within the RCB. The horizontal spring represents the top and bottom rebars, and the shear (vertical) spring represents the shear stiffness against vertical relative displacements across the crack. The behavior of the mat during

earthquakes is influenced by the characteristics of these springs, and cannot be defined without them. The difficulty in obtaining proper values for these springs is discussed below.

Assume that the crack is wide and there is no contact between concrete surfaces across a crack. The total shear force at the cracked section can only be transmitted across the crack by dowel action of the top and bottom rebars alone. Both shear strength and stiffness for this type of shear transfer mechanism are only a small fraction in comparison with that of an uncracked section. Furthermore, this dowel action will cause longitudinal splitting of the concrete along the horizontal bars and lead to further loss of bond and consequent stiffness degradation in both shear (vertical) and horizontal extension. Since the vertical rebars (stirrups) do not cross the cracks, they do not participate directly in carrying shear, but contribute in confining the longitudinal splitting phenomenon. The actual shear stiffness of the cracked mat lies in between a condition of an uncracked section and a total separation at the crack without any friction force. It is very difficult to determine a proper shear stiffness for the cracked mat. The crack width is unknown, and there is no test data available on shear stiffness for a 12-foot deep member. There is however, some test data of small specimens. It is difficult to extrapolate from these data because there are indications that the crack patterns are different between a small specimen and a deep member due to depth effect.

The horizontal extension spring in the elastic range can be calculated mathematically as AE/L, where A is the total steel area, E is the modulus of elasticity of steel, and L is an effective length. Theoretically, the effective length is the crack width between two solid blocks linked by the steel. Some stress analysts argue that the horizontal extension spring may be assumed as a hinge since the crack

width is small; mathematics of this analysis would indicate no problem at all in transmitting all horizontal forces from one side of the crack to the other side due to this hinge assumption. However, in reality the behavior may be different. A lesson learned from the 1971 San Fernando earthquake (a lesson which was obvious but not readily susceptible to stress calculation) was that bridges pull apart or suffer damage due to hammering effect at hinge expansion joints. The California Highway Department has since initiated a broad program to tie and strengthen all the hinge expansion joints.

From the above discussion, it may be concluded that the response of the cracked mat and its superstructures during earthquakes will deviate from what was originally predicted when uncracked sections were assumed. Whether the deviations are significant enough to impair the safe shutdown ability during a safe shutdown earthquake (SSE) is not yet known.

In connection with the shear displacement problem, I expressed my concern that vertical through-cracks might have occurred and its undesirability of this effect in a letter dated October 17, 1983 to the applicant. The applicant responded in a letter dated October 31, 1983, that such a postulation was unreasonable and that there was no direct evidence to support it, but acknowledged that "The formation of vertical or near vertical through-cracks would imply an occurrence of shear failure in the mat concrete." It appeared that the applicant was aware of the significance of the through-cracks. However, no response in this regard has been received since the NDT.

IV. EVALUATION OF BNL REPORT AND ITS ADDENDUM

IV.1 Summary of BNL Report and Its Addendum

After reviewing the BNL report and its addendum, I was left with an understanding as follows:

The dead loads (weights) were mainly responsible for producing the extensive cracks both in length and depth during a period before the backfill soils were placed. Many of the reinforcing steel bars had yielded during that construction period and it was the backfill soils that closed the cracks that developed during construction.

BNL performed an approximate analysis for a mat strip in the N-S direction across the center of the reactor, the same strip as shown in Fig. 1a of this report. The strip was subjected to dead loads taken from the HEA computer input. BNL stated the result of its analysis: "the maximum (bending) moment for this case occurs close to the center of the reactor and results in tension on the top surface of the mat" (BNL report p. 11). BNL used this analysis result to indicate that cracking could have occurred during construction due to dead loads.

Regarding cracks outside of the ringwall, BNL selected several bending moments from the HEA reports and stated that "....the dead load portion of the HEA finite element analysis provide a reasonable simulation of the actual loading to permit a conclusion as to the probable cause of the surface cracking" (BNL report p. 11).

BNL also performed an approximate analysis on the stresses developed as a result of concrete block construction sequences. BNL stated "..... even for rather dramatic variations in foundation moduli, only a minimal concrete strength is required to prevent [misspelled as present] either a shear or moment crack" (BNL report p. B-4 of Appendix B). As assessed by BNL this indicates that the stresses induced during the concrete block construction sequence, which was not included in the original EBASCO design, was insignificant.

BNL identified and recommended that detailed confirmatory calculations be performed for five subjects, but stated that "Based upon our approximate calculations together with engineering judgement, we do not anticipate that the refinement of these analyses will lead to major changes in calculated stress level." This indicates a calculation accuracy check only. BNL concluded that the safety margins in the design of the base mat are adequate.

After BNL learned the preliminary NDT results, indicating extensive cracking both in length and depth, it advanced a domino top rebar yielding phenomenon to explain the existing crack formation process and fixed the neutral axis as 18 inches above the bottom face of the mat in bending to further support the extensive depths of the cracks. Regarding the effect of cracks, BNL stated that cracks did not affect the bending moment and shear capacities. BNL concluded that the cracked mat was adequate.

IV.2 Evaluation of BNL Report and Its Addendum

IV.2.1 Adequacy of BNL Analysis

The BNL analysis result, as mentioned in the previous section, was a shock to me. It not only contradicted my intuition on the deflected shape of the mat, but also contradicted the analysis result presented by both EBASCO and HEA. In order to produce tension on the top surface of the mat under the reactor, it would require that portion of the mat to bend into an inverted dish type from an assumed initial flat position. Theoretically, it may be possible to produce such a shape with much heavier loads than the reactor acting outside of the ring wall with a particular arrangement for the mat stiffness and soil stiffness. However, with the heavy weight of the reactor acting downward and less heavy weight outside the ring wall, the deflected shape of the mat under

the reactor should be a dish type, as shown in Figures 1b2 and 1c2. EBASCO's computer analysis indicates compression at the top, as shown in Figures 2 and 3. The computer plot of the dish type deflection under dead loads was presented in Appendix E of HEA Report No. 2. I must conclude that the BNL analysis result is incorrect.

IV.2.2 Adequacy of BNL Interpretation of HEA Data

Table 1 (page 5) of the BNL report contained selected bending moments of the mat from HEA computer data. The areas between the ring wall and side walls along an E-W strip across the center of the reactor is represented by element numbers in Areas R-P-2M-1A and R-P1-12A-9M. N-S bending moments under the heading My in Area R-P1-12A-9M are four negatives, indicating compression at the top of the mat according to the legend, and two positives with magnitudes of 440 and 428 ft-K/ft. In Area R-P-2M-1A, the maximum positive bending moment is 670 kip-ft/ft. Actually, the bending moments in Areas R-P-2M-1A and R-P1-12A-9M are supposed to be the same as those bending moments of about 200 ft-k/ft for variable soil modulus between the ring wall and side walls, as shown in Fig. 2 of this report. Although there is a difference in magnitudes of bending moments between the EBASCO and BNL presentation for a supposedly identical value (EBASCO has taken out the 50 per cent increase of dead loads that may be retained by HEA input), the required cracking bending moment of 1810 ft-k/ft, as calculated and shown in Figure 7, is much greater than any of these acting bending moments. This certainly raises a question as to the BNL assessment that the dead load was mainly responsible for the cracks.

IV.2.3 The Domino Phenomenon of Top Rebar Yielding

Assume that the mat is only reinforced in the N-S direction with no rebars in the E-W direction. When steel stress in a single narrow strip is raised beyond its yield strength by a bending moment, this strip becomes more flexible and allows the adjacent rebars to carry any additional bending moments. This spreading phenomenon of rebar yielding laterally, as described in the BNL addendum, has been confirmed by physical tests, and I have personally witnessed such phenomenon in cantilever plates that I have tested. However, the mat is not reinforced in the N-S direction only. The mat is actually reinforced in both directions. Any perpendicular strips must deflect under load the same total amount at their common point; hence, they share the load and bending moments in both directions caused by the load. If concrete is cracked in the N-S strip, and the E-W strip is uncracked, then a greater portion of the total load and bending moments will be taken by the uncracked E-W strip. The mat, when reinforced in both directions, has the capability of redistributing the load and bending moment in both directions. This capability, coupled with the capability of transferring load and bending moment laterally, makes the mat one of the most ductile structural elements against bending moments. The BNL statement the "Since such a failure is rather abrupt, one would expect the cracks to propagate to deeper depths..." is contrary to the two-way reinforcement condition of the mat, and thus its behavior.

Rebars will not yield unless the magnitude of a bending moment is sufficiently large to raise its stress beyond yield strength. Information provided by EBASCO and HEA does not indicate that such a bending moment exists, nor did BNL provide that information. The bending moment required to initiate the yielding of top rebar is a little less than 2093 ft-k/ft as shown in Figure 8, while the actual negative bending moment was on the order of 100 ft-k/ft. The top rebar did not yield due to dead loads, as the analyses of EBASCO and HEA have

indicated, and perhaps would not yield even if analyses considered the actual soil conditions during construction were performed.

Furthermore, had the top rebars yielded to the extent, as described in the BNL addendum, the wide crack widths should have been noticed by the field personnel during construction.

IV.2.4 The Location of a Neutral Axis

The magnitude of a bending moment required to initiate concrete cracking on the top surface of the mat and its corresponding position of the neutral axis are shown in Figure 7. The magnitude is 1810 ft-k/ft and the position of the neutral axis is 2 inches below the mid-depth. Figure 8 shows a magnitude of bending moment of 2093 ft-k/ft corresponding to a neutral axis of 18 inches above the bottom face of the mat. Where the neutral axis actually lies depends on both the magnitude of a bending moment and geometry of a cross section and properties of materials. Thus, the location of a neutral axis is neither a constant value nor independent of the bending moment acting on the cross section, unlike the BNL report implied.

For an under-reinforced concrete cross section (steel yields before concrete being crashed), an increase of an acting bending moment will push the neutral axis toward the compression side to maintain a balance of a tension force and a compression force at the opposite side of the neutral axis. For an over-reinforced concrete cross section (concrete being crashed before steel yields), the neutral axis moves toward the tension side with an increase of bending moments to provide a larger area of concrete for compression.

In order to push the neutral axis down to 18 inches above the bottom face of the mat as stated in the BNL addendum, it requires a magnitude of negative bending moment of 2093 ft-k/ft to act on the cross section. This magnitude is about 20 times the magnitude of a negative bending moment generated from the mat with full dead loads and no backfill soils at all with an assumed soil modulus of 150 pound per cubic inch, as shown in Fig. 2 of this report.

IV 2.5 Detailed Confirmatory Calculations

BNL recommended that detailed confirmatory calculations be performed for the following five subjects:

a. dynamic coupling between the reactor building and the base mat for seismic stresses resulting from the vertical earthquake input.
b. dynamic effects of lateral soil/water loadings.
c. artificial boundary constraints in finite element models.
d. fineness of base mat element mesh.
e. origin of cracks in the vertical walls.

On subject (a), BNL stated, "Since the effects are localized, we believe that they should not greatly influence the gross resultant forces acting on the mat" (BNL report p. 16). On subject (b) BNL stated, "However approximate estimates of these dynamic effects made by BNL indicate that the total lateral load should change by no more than 15 percent" (BNL report p. 17). There are no estimates of deviation given on the remaining three subjects. According to the BNL statement, "Based upon our approximate calculations together with engineering judgement, we do not anticipate that the refinement of these analyses will lead to major changes in calculated stress levels," the purpose of the detailed calculations is striving for calculation accuracy, not for errors, such

as may result from major incorrect assumptions or mathematical representations.

Aside from the expensive computer cost, the value of these detailed calculations recommended by BNL is questionable. I pointed out in Section III.4 that a decrease of soil stiffness from 150 pounds per cubic inch to 70 pounds per cubic inch within the ring wall resulted in a positive bending moment increase from about 200 ft-k/ft to 800 ft-k/ft. This type of 400 percent increase in bending moments is certainly not covered in the detailed calculations recommended by BNL. Furthermore, the assumptions made in soil stiffness for the two Ebasco analyses that led to the 400 percent difference in bending moments do not represent the actual soil conditions during the initial construction stages, but are applicable to current and future soil conditions. In fact, the distribution of soil stiffness during the initial construction stages was stiffer in the middle and less stiff at sides in the N-S direction of the mat according to the construction sequence. Structural analysis has not been performed for this type of soil conditions which may be responsible for the cause of concrete cracking on the top of the mat.

Another analysis, which has not been performed, would add to our understanding on whether or not the temperature generated due to cement hydration was high enough to have been the most likely cause for the cracks in the mat during its cooldown period.

The basic attitude of striving for accuracy is commendable, but it is not justifiable or necessary to require such a level of accuracy, such as 15 percent, while another effect, which was not considered by BNL, has shown a 400 percent variation and would be even greater if the actual soil stiffness during the initial construction stages was considered.

IV.3 Effect of Cracking

BNL stated that the concrete cracking would not affect the bending moment and shear capacities of the mat. I agree with the statement on bending moment capacity, but disagree with the statement on shear capacity. In fact, it is a well known phenomenon that cracking reduces shear capacity. "A pre-existing crack along the shear plane will both reduce the ultimate shear transfer strength and increase the slip at all levels of load" is the first conclusion in a paper entitled, "Shear Transfer in Reinforced Concrete With Moment or Tension Acting Across the Shear Plane," by A. H. Mattock, et al., PCI Journal, March-April 1972.

IV.4 Standard For Acceptance

BNL's report, dated July 8, 1984, concluded that the mat was adequate before the extensive cracking was known. BNL's addendum, dated August 3, 1984, concluded that the mat was also adequate with a knowledge of three primary cracks in the mat under the reactor in the E-W direction with unknown crack widths. It is my understanding that BNL concluded again that the mat was still adequate when it learned that there were 7 cracks with unknown crack widths under the reactor. However, BNL did not indicate the acceptance standards for cracking that it used or its basis or reasoning for accepting the adequacy of the cracked mat for each of the three conditions, and instead relied on its judgement.

IV.5 Conclusions

After reviewing the mat design and its construction sequences, I have concluded that the BNL postulation regarding concrete cracking and the many top reinforcing steel bars yielding which occurred due to dead loads (with an assumed soil stiffness of 150 pci or the variable soil

moduli) is incorrect. BNL statement that concrete cracking does not affect shear capacity is also incorrect. BNL should state its standards or reasons for accepting such an extensive cracking both in length and depth.

V. CONCLUSION AND RECOMMENDATION

V.1 Introduction

Since the submittal of my earlier affidavit on November 28, 1983, three important issues have been raised with respect to Waterford 3 base mat: (1) the construction quality control problem, (2) the nonuniform nature of soil problem, and (3) the extensive concrete cracking problem.

The construction quality control problem as related to the design, such as concrete quality, bond between rebars and concrete, and force transfer at the ends of two rebars through Cadwelding, has been resolved and documented in a report by Mr. R. E. Philleo (Reference 3). However, the other two issues have not yet been resolved. The nonuniform nature of soils under the mat was discovered and reported by Dr. J. Chen (Reference 2), but this view has been consistently disputed by BNL. This dispute contributed to the difficulties in identifying whether the nonuniform nature of soils was responsible for the concrete cracking and to what degree. No analysis taking into consideration the nonuniform nature of soils has ever been performed, and, thus, the structural effect on the mat is unknown.

Concrete cracking, besides its effect on structural responses during earthquakes, has been qualitatively discussed in Section III; its effect on the structural integrity via steel corrosion, durability problems, and Code limitations on crack widths and comparisons are described. Two practical repair methods often used to deal with crack problems are mentioned. My views on the adequacy of the mat and recommendations are presented.

V.2 Consequence of Concrete Cracking - Steel Corrosion and Durability Problems

Reinforcing steel does not corrode when it is tightly bonded to good quality concrete. This is because a protective oxide coating forms around the steel in the highly alkaline environment of concrete. When oxygen, moisture, and aggressive ions enter the concrete through cracks, they may reduce or even destroy the alkaline environment and cause steel corrosion. Corrosion of steel creates iron oxides and hydroxides and thus increases the steel volume. The increase of the steel volume produces high bursting stresses around the rebar and further creates radial cracking in the concrete. A delaminated surface resulted from rebar corrosion is a well-known phenomenon and problem in concrete bridge decks.

Steel corrosion and durability problems are well known. However, the solutions to these problems are often expensive and difficult. The concrete industry has preferred preventive approaches to eliminate or reduce the possibility of occurrence of these problems through crack control. Effective crack control measures are important for the durability of concrete structures by eliminating potential for rebar corrosion and concrete disintegration.

V.3 Code Limitations on Crack Widths and Comparison

The ACI Building Code (Reference 4) indirectly sets the maximum crack widths of 0.016 and 0.013 inches for interior and exterior exposure conditions, respectively. The AASHTO Specification (Reference 5) indirectly sets the maximum crack width of 0.012 inches. The CEB-FIP Recommendation (Reference 6) specifies an allowable crack width of about 0.008 inches for exterior exposure condition. The ACI Committee 224 (Reference 8) recommends permissible crack widths versus exposure conditions as follows:

4.3 - Permissible crack widths versus exposure conditions in reinforced concrete

Table 4.1 is a general guide for recommended limits of tolerable crack widths at the tensile face of reinforced concrete structures. The values given are based on a summary interpretation of Table 1 in Reference 4.3, and are recommended by ACI Committee 224 as maximum values.

TABLE 4.1 - PERMISSIBLE CRACK WIDTHS, REINFORCED CONCRETE

Exposure condition	Maximum allowable crack width, in*
Dry air or protective membrane	0.016
Humidity, moist air, soil	0.012
Deicing chemicals	0.007
Seawater and seawater spray; wetting and drying	0.006
Water retaining structures	0.004

*0.001 in. = 0.025 mm.

In comparison with these values, ACI 318-71 Section 10.6 considers two sets of exposure conditions only: w = 0.016 in. for interior exposure, and w = 0.013 in. for exterior exposure.

It must be pointed out that the crack widths mentioned are measured at the tensile face, which is supposed to be the maximum crack width of the section, while the measured surface crack widths on the mat in Section III.5 is on the compression face, which is supposed to be the mimimum crack width of the section.

For deep concrete members without longitudinal reinforcing bar in the mid-portion, as is the case of the mat, the crack width in the mid-depth could be substantially greater than that at the main tensile steel. This phenomenon and its associated durability problems are mentioned in Enclosures I through III.

Since the mat is a mass concrete, concrete cracking is of importance. The definition of mass concrete is defined by ACI Committee 207 (Reference 7) as "any large volume of cast-in-place concrete with dimensions large enough to require that measures be taken to cope with the generation of heat and attendant volume change to minimize cracking." The importance of cracking in mass concrete is excerpted "the primary requirement involved in mass concrete construction is that the completed structure is a monolithic mass that is free from cracks..." from a paper written by a distinguished engineer and past-president of ACI; the first page of the paper is shown in Enclosure IV.

Considering the many through-cracks, the extensive length of cracks, the unknown crack widths within the RCB, the comparison of the existing crack widths to that of Code limitations, the potential steel corrosion

and concrete durability problems, the important effect of cracking on the primary requirement of a monolithic mass concrete, and the uncalculated dynamic responses as mentioned in Section III.6, there has not been enough evidence to conclude that the existing cracks can safely be ignored.

V.4 Two Practical Repair Methods

Two types of repairs are often used. Pressure grouting or epoxy injection to fill crack spaces is one type to restore the bond in concrete that was lost due to cracking. ACI Committee 224 (Reference 9) reported that cracks as narrow as 0.002 inches can be bonded by the injection of epoxy. Another type of repair is to use prestressing tendons to tie the crack faces together, which strengthens the weak plane in resisting tensile and shearing forces.

V.5 Adequacy of the Mat

Successful crack repair with a monitoring program to assure the continued successful performance of the repaired mat would be sufficient and no further analysis would be needed. This is because the possible major contributing factors to the cause of the cracks would have vanished and would not appear again. Thermal stress due to the cement hydration process, which might have produced the cracks, would not appear again. Stress resulting from concrete block construction sequences has leveled off. Stress associated with differential settlements decreases as the settlements of soils became stabilized through soil consolidation process. The significant groundwater level changes during construction would not reappear.

The philosophy of this approach is to acknowledge our inability at present to fully understand and explain what kind of abuses the mat has withstood and to abandon our search for the past history of the mat, and is based on a verified assurance that the mat was adequately designed for current and future soil conditions.

If the mat is not repaired, further evidence is needed to prove it is functional during earthquakes and durable during the life of the plant.

V.6 Recommendation

Since the plant is at the stage of obtaining an operating license, the mathematical analyses, which were appropriate in April when we first recommended, are not now timely. Considerations should now be given to practical repairs rather than mathematical analysis. It seems that a repair is the most reliable and realistic way to eliminate potential problems to the mat integrity and plant safety. I believe that the initial investment for repairs is cost effective compared to the increase of durability of the mat and of safety of the plant. Should the repair be found infeasible or not cost effective, other means such as mathematical analysis can still be used.

VI. REFERENCES

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2. "Geotechnical Engineering Safety Evaluation, Waterford No. 3, Louisiana Power and Light Company," John T. Chen, May 1, 1984.

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4. ACI Committee 318, Building Code Requirements for Reinforced Concrete), ACI 318-83), American Concrete Institute, Detroit, MI, 1983.

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6. Recommendation International CEB-FIP, 1970, Comite Europeen du Beton - Federation Internationale de la Precontrainte, Paris, 1970.

7. ACI committee 207, "Mass Concrete for Dams and Other Massive Structures," ACI Journal, Proceedings V. 67, No. 4, April 1970, pp. 273-309.

8. ACI Committee 224, "Control of Cracking in Concrete Structures," Journal of the American Concrete Institute, No. 12, Vol. 69, December 1972.

9. ACI Committee 224, "Causes, Evaluation, and Repair of Cracks in Concrete Structures," ACI Journal, Proceedings V. 81, No. 3, pp. 209-316, May-June 1984.

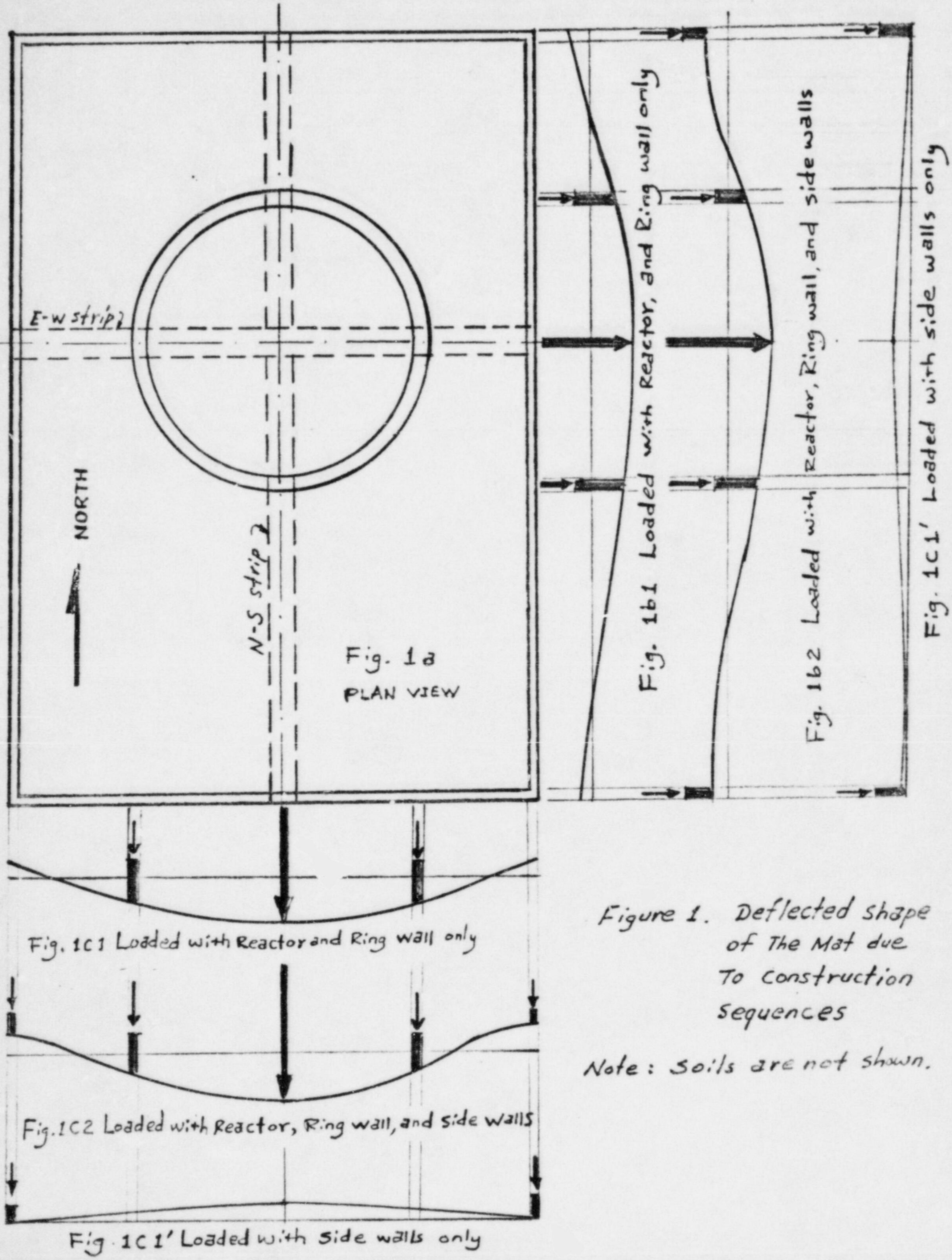

Figure 1. Deflected shape of The Mat due To construction sequences

Note: Soils are not shown.

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BY C G LEE DATE 11-19-84 SHEET 3 OF 7

CHKD. BY T. Yang DATE 11-19-84 OFS NO. ____ DEPT. NO. ____

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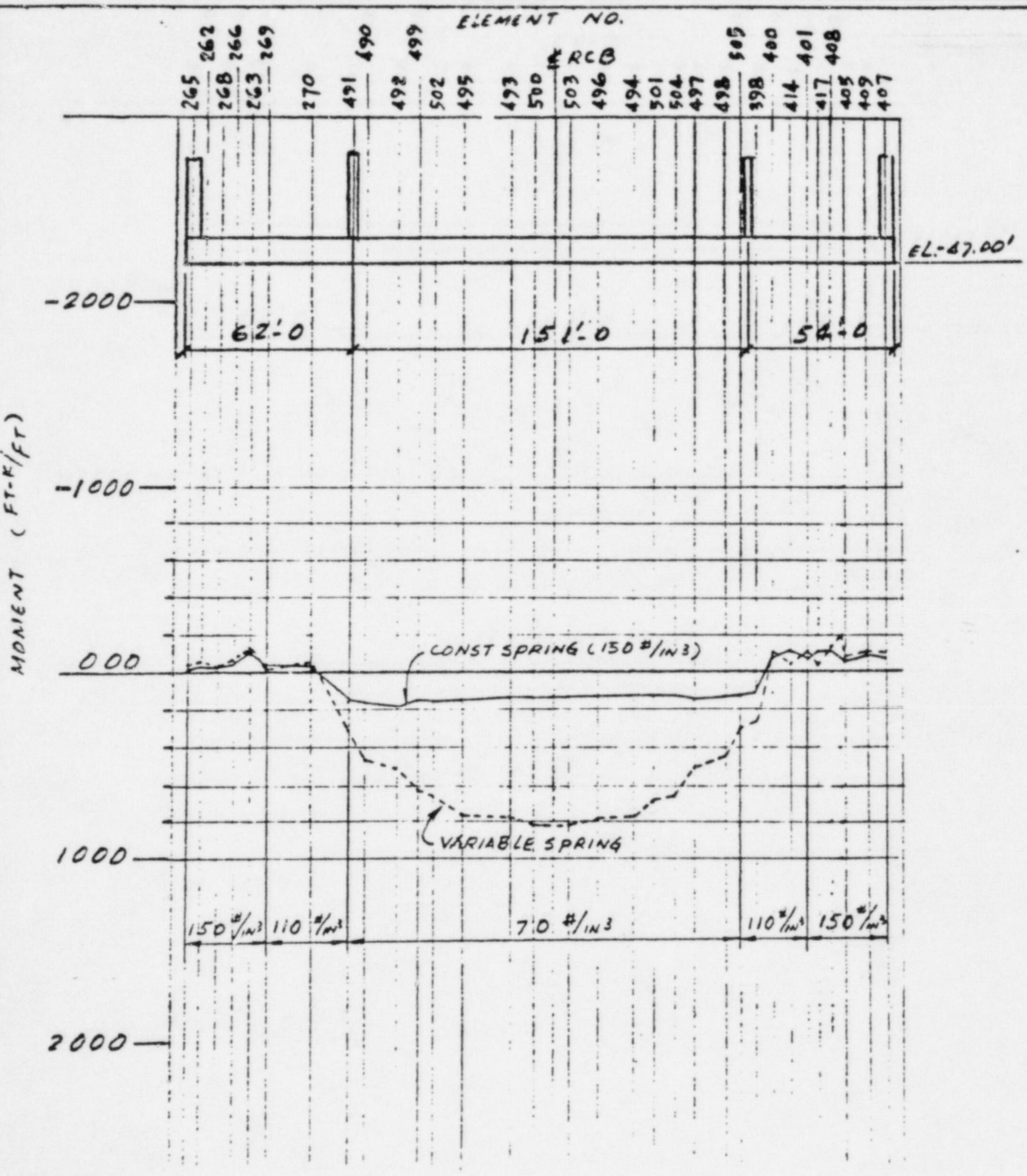
PROJECT WATERFORD STEAM ELECTRIC STATION 1977 1165 MW INSTALLATION - UNIT 3

SUBJECT MAT MOMENT CURVES



LEGEND

———— CONST MODULUS

- - - - - - VARIABLE ,,

Fig. 2 MOMENT CURVE - SECT A-A (NS BENDING)

D (DEAD LOAD) ONLY

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CHKD. BY C G LEE DATE NOV. 19. 84

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PROJECT WATERFORD STEAM ELECTRIC STATION 1977 1165 MW INSTALLATION - UNIT 3

SUBJECT MAT MOMENT CURVES

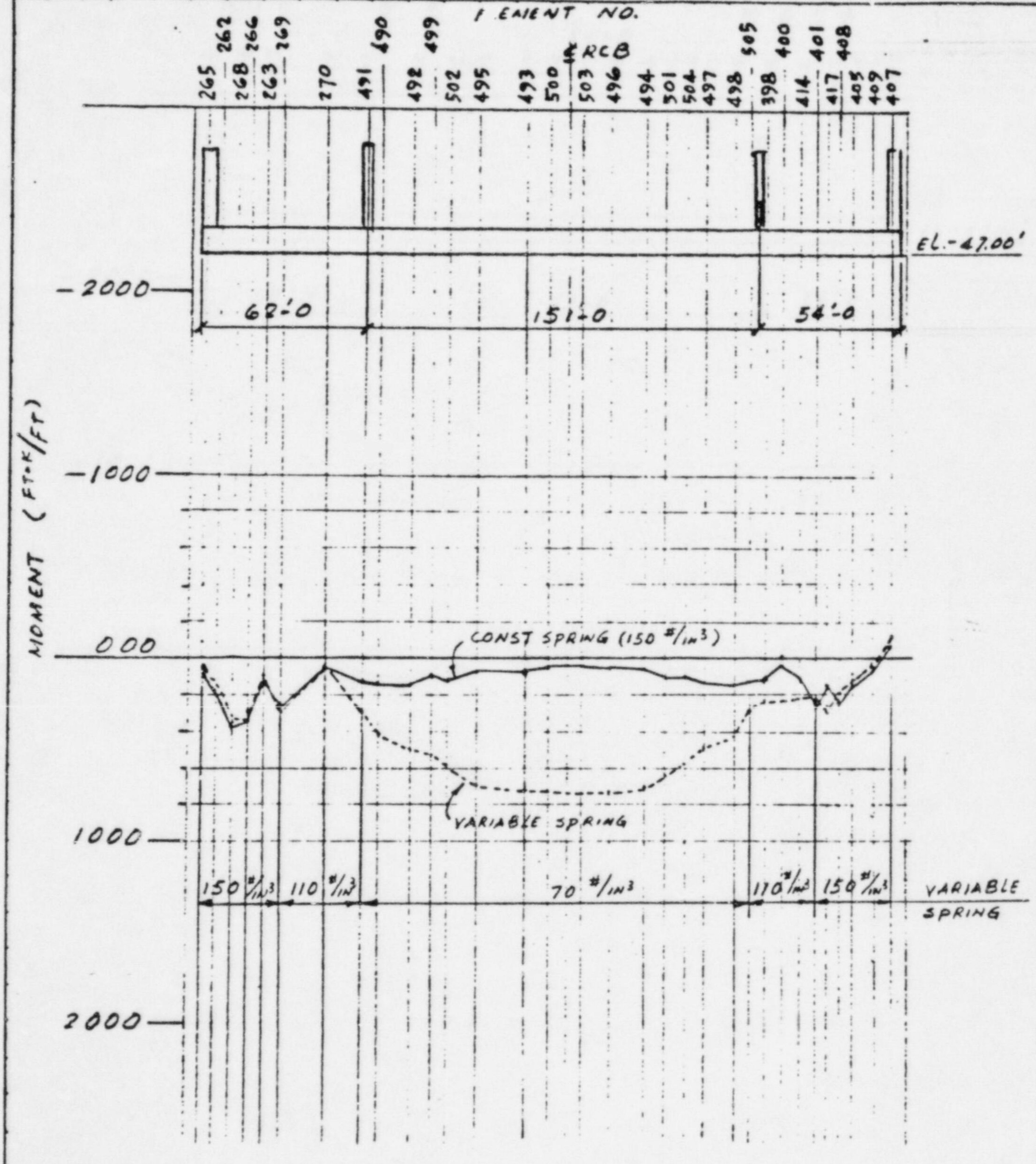

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Fig. 3 MOMENT CURVE - SECT A-A (N-S BENDING)

NOR OPER $C = 1.5(D+L') + 1.8(L+S') + 1.0B$

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PROJECT WATERFORD S.E.S. UNIT NO.3

SUBJECT COMMON FDN MAT - REINF

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BY CGLEE DATE 11-27-84

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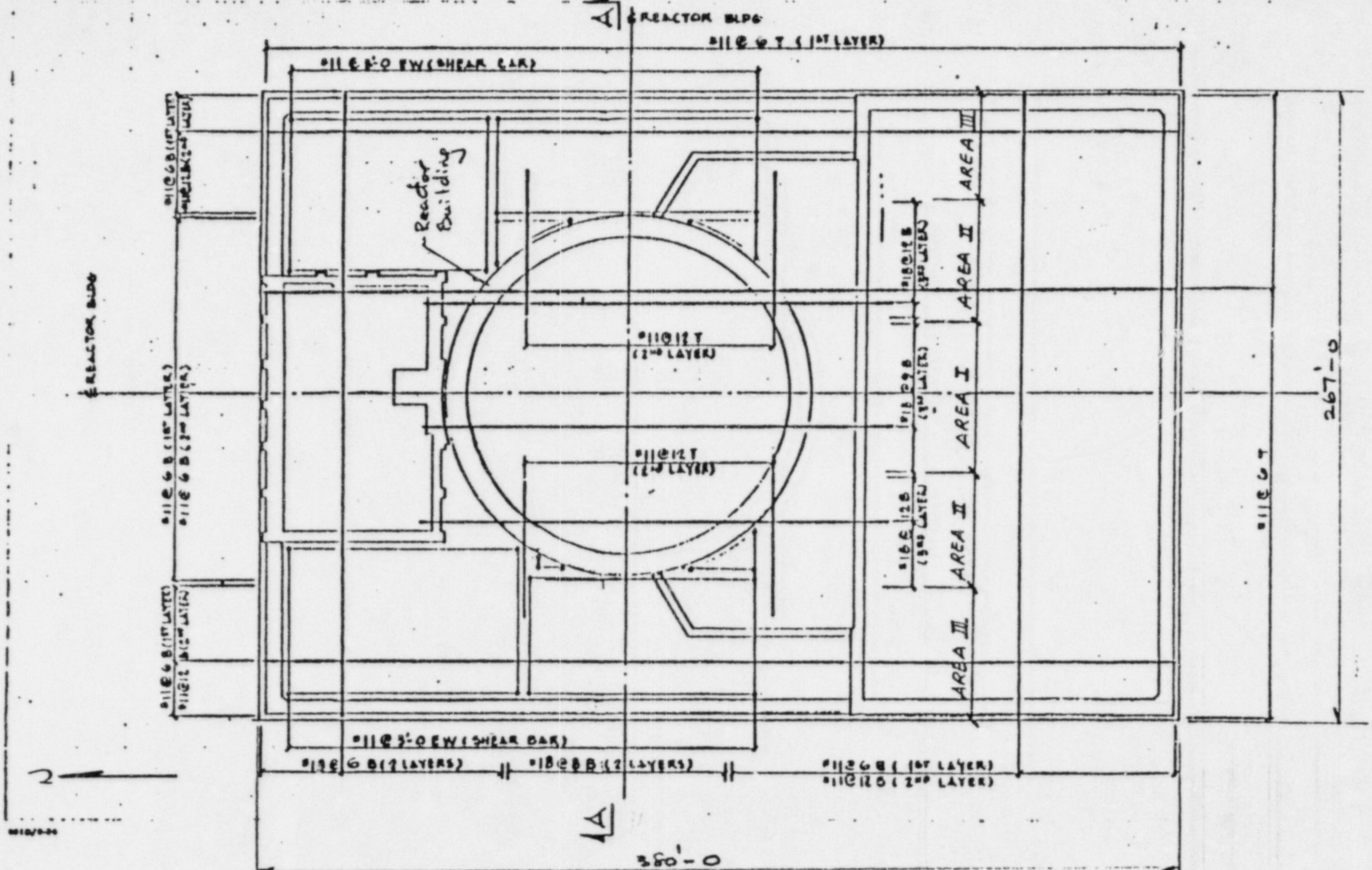
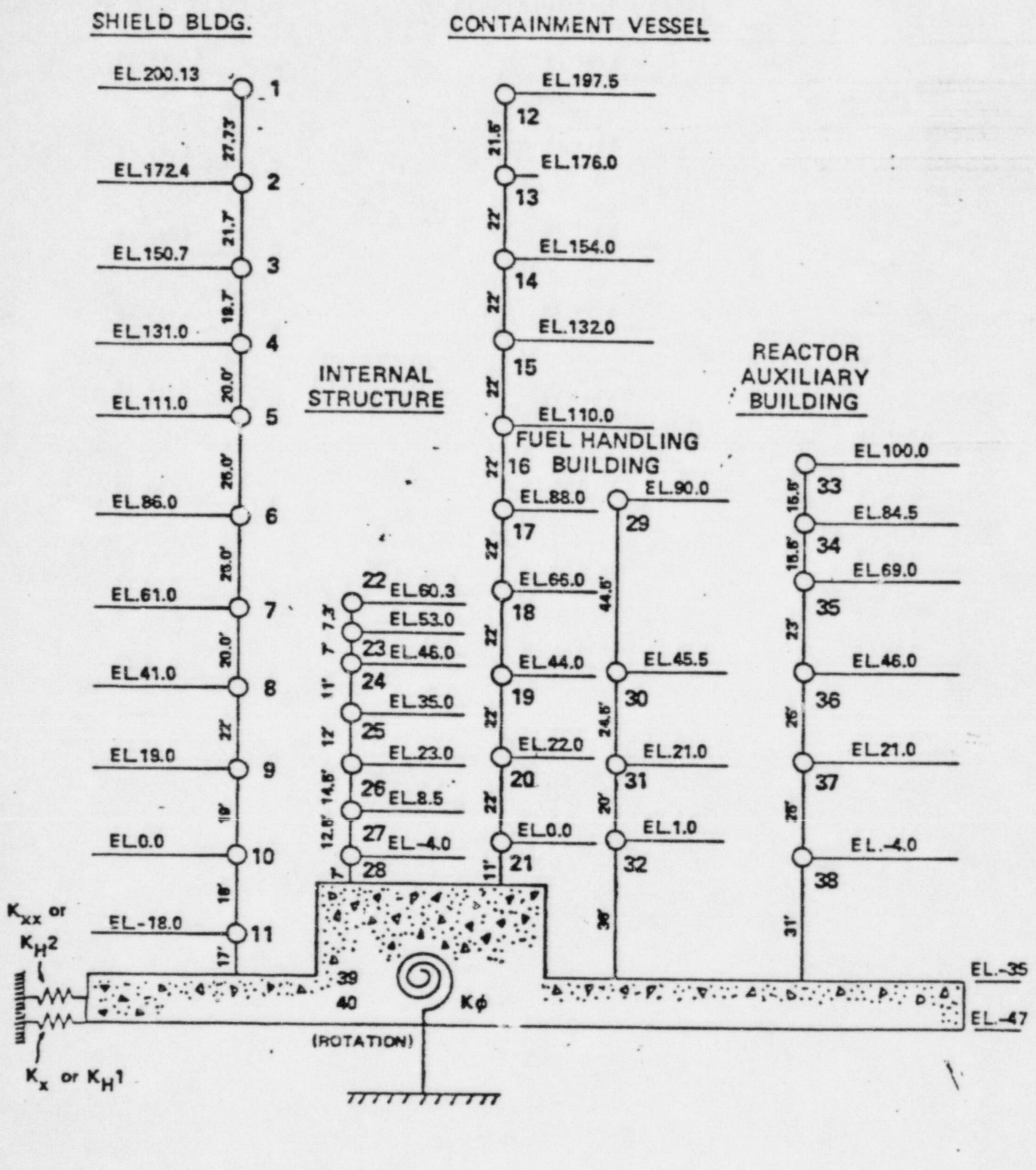


Figure 4 Reinforcing steel of the Mat



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1977 1165 MW INSTALLATION - UNIT 3

AMENDMENT NO. 2 (3/79)

LOUISIANA POWER & LIGHT CO. Waterford Steam Electric Station	Fig. 5 MATHEMATICAL MODEL FOR SEISMIC ANALYSIS (HORIZONTAL EXCITATIONS)	Figure 3.7-9

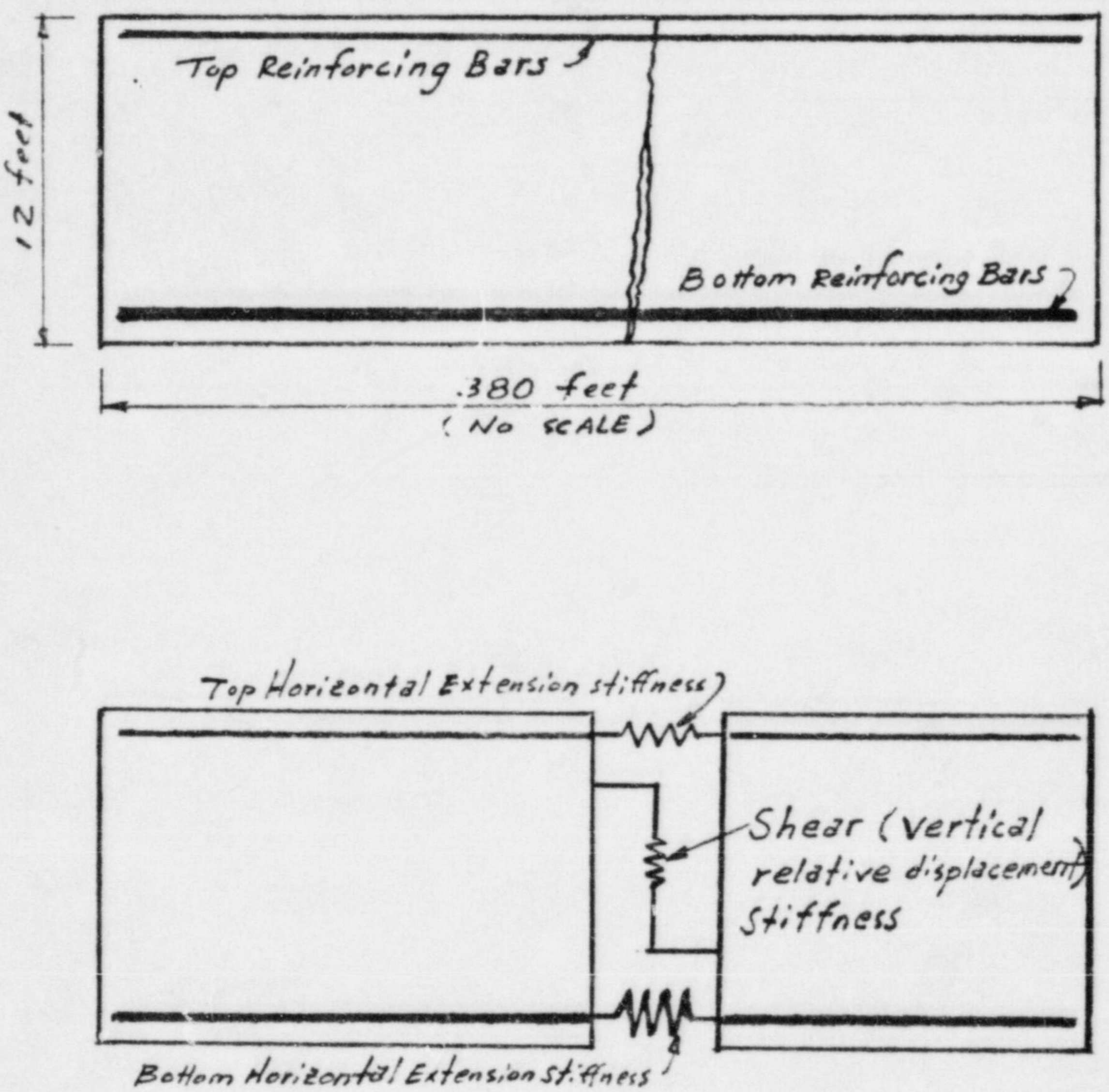

Figure 6. The Cracked Mat And Its Mathematical Model

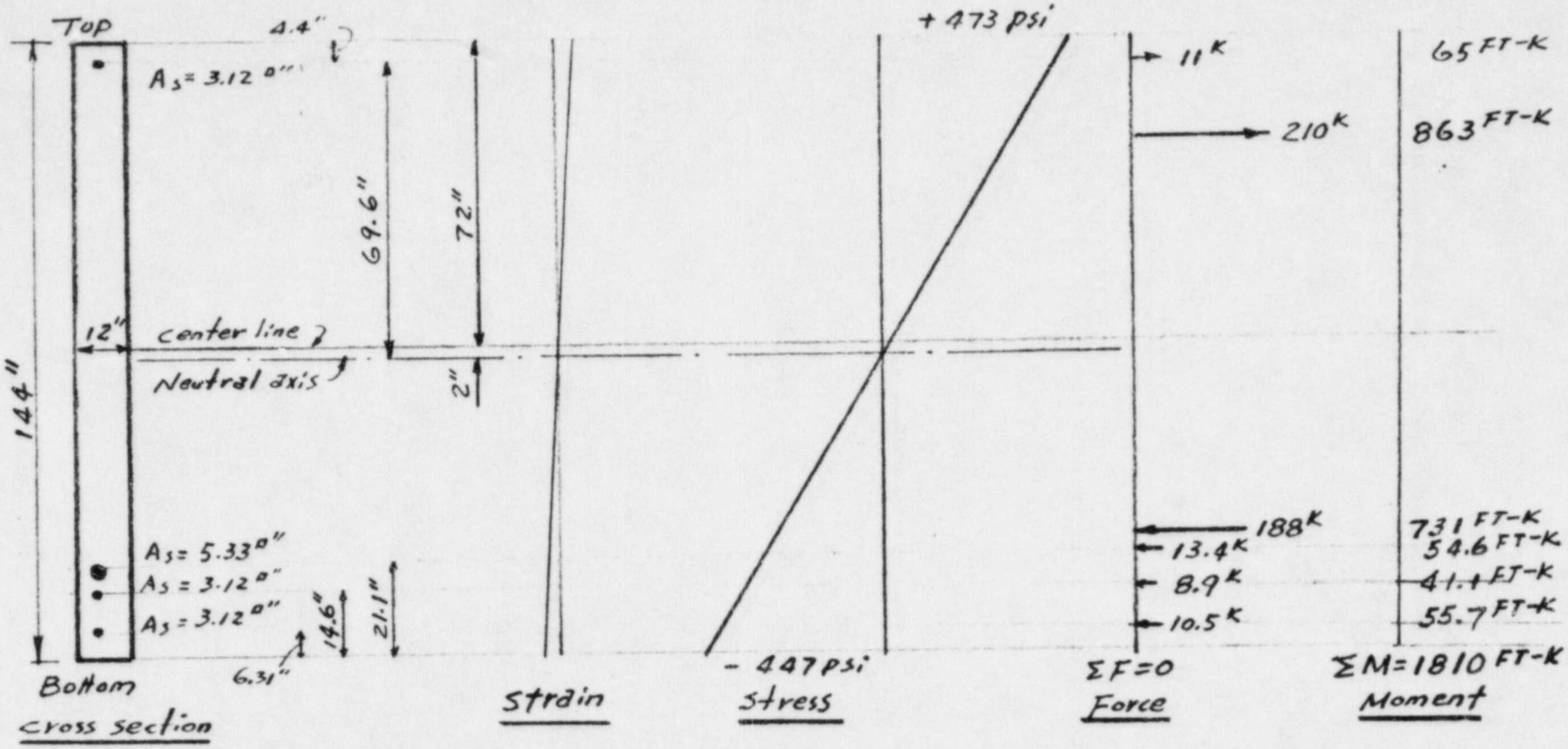

Figure 7. Cracking Bending Moment AND Its Corresponding Location of Neutral Axis IN THE N-S Direction.

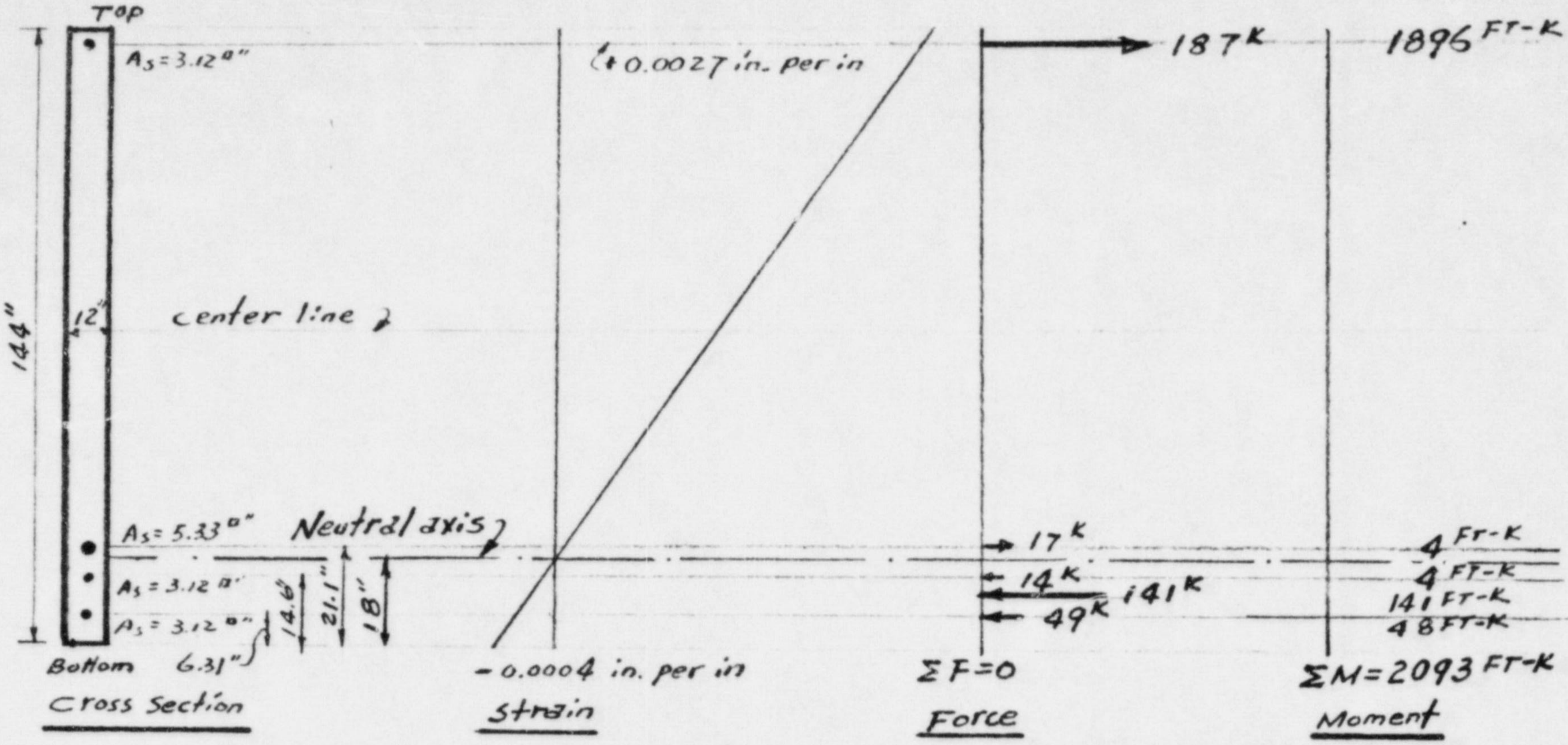

Figure 8. The Bending Moment Corresponds To A Neutral Axis Located At 18 inches Above The Bottom Face of The Mat In The N-S Direction.

Enclosure I

SUMMARY

Several large reinforced concrete highway bent cap girders, designed according to ACI and AASHTO provisions, were found to have very wide cracks near middepth on the side faces. Although the crack width at the level of main reinforcement was acceptable, the side face cracks near middepth were up to three times as wide. This indicated potential durability problems. A 3/8 scale laboratory model using deformed bars and reduced maximum size aggregate accurately reproduced the crack pattern and crack widths of the full size bent caps. A simplified test specimen was developed to accurately simulate the behavior of a portion of a beam under constant moment loading. A series of 44 specimens investigated the variables affecting side face cracking: amount and distribution of side face reinforcement, cover, web width, and beam depth. A relatively simple two-dimensional finite element analysis generally confirmed the laboratory results. A new design procedure was developed to control side face crack widths and was simplified for code use. To verify the new design procedure, the original model bent cap with the serious side face cracking problem was redesigned and tested. The procedure worked very well. Although the new procedure requires substantially more side face reinforcement for large beams than present provisions do, it appears that the side face cracking problem can be controlled at little or no additional cost by considering the flexural strength contribution of the side face reinforcement.

"Control of Cacking on the side faces of Large Reinforced Concrete Beams" by G.C. Frantz and J.E. Breen
Research Report 198-F Project 3-5-76-198
Cooperative Highway Research Program with Texas
State Department of Highways
and
U.S. Department of Transportation
Federal Highway Administration
THE university of Texas at Austin September 1978

Enclosure II

tables unless it happens to be a shape commonly used in precast work or the like. The necessary A_s for a beam such as that of Fig. 3.16 can be found by a cut-and-try process that travels over much of the same ground used in Section 3.15 for finding the allowable design moment to accompany a given A_s. The procedure would be as follows:

1. Assume z, noting that with the narrow top width in Fig. 3.16 N_c acts farther from the top than in a rectangular beam of constant width.
2. Calculate $A_s = M_n/(f_y z)$.
3. Evaluate depth of stress block from $N_{cn} = N_{tn}$, possibly first finding A_c in compression from $A_c = A_s f_y / 0.85 f'_c$.
4. Find centroid of A_c to get location of N_{cn} and a better z.
5. Recycle (if the original estimate of z was poor), to the accuracy desired.

An alternate procedure for step 4 would be to break down A_c into simple pieces, as done for the T-beam in Fig. 4.6, each with its individual A_s and couple. This does not, however, so clearly indicate how much error was in the assumed z of step 1 and what would be the better guess for the next cycle.

4.9 FACE STEEL ON LARGE BEAMS

The primary Code emphasis on deep beams is for relatively short spans, ℓ_n not more than $5d$. (See coverage in Sec. 5.2.1 as primarily a shear problem.) However, Code 10.6.7* relates primarily to flexural cracking in any web more than 3 ft deep, regardless of span length, even in constant moment zones where shear is essentially zero. It calls for longitudinal reinforcement "at least 10 percent of the area of the flexural reinforcement—placed near the side faces of the web and distributed in the zone of flexural tension."

A depth of more than 36 in. is not a common beam size for buildings, but it is common for bridge structures; this specification is written to control flexural cracking on the deep side faces. Unfortunately, it is *not* adequate for this purpose. Actual construction and tests in France and at the University of Texas at Austin show this inaccuracy, although the final report on the latter is not yet issued.

The basic problem is simple, once it is noted. Ordinary beams are designed on the assumption that the strain gradient from flexure is linear, the shear strains being negligible in shallow members. In the deep beam

*The highway bridge code (AASHTO) discussed in Chapter 17 is essentially the same in this respect.

"Reinforced Concrete Fundamentals" by Phil M. Ferguson, 4th edition, John Wiley & Sons, Inc. 1979

the flexural strains, top and bottom, are about the same as in a shallow beam, which means the flexural strain gradient varies essentially inversely with depth, thus very small in very deep members. The shear strains are no longer negligible in comparison. Instead they can almost totally cancel the flexural strains and relax the longitudinal tension in the concrete just above A_s and widen the flexural crack once it gets well above A_s.

A second pattern, less predictable except from experiments, is that all the cracks forming at the level of the bars do *not* rise to equal heights in the web. With deep beams without horizontal web bars there may be 3 or 4 cracks at the A_s level for each crack continuing farther up in the web. In this way also the longitudinal strains can lead to web crack widths that may be several times as wide as those at the A_s level.

The function of longitudinal face bars in the web is to stretch the concrete (help it carry some tension) and also to cause more web cracks of narrower average width, rather than fewer wide cracks. The author was almost startled by the flexural crack widths in the first 36 in. beam he tested, cracks substantially wider at mid-depth than at A_s level.

Tests in France[3] reported in 1972 on beams 1 m deep and 0.3 m thick seem to show that bars totaling 0.25 percent of the web area and placed between A_s and the neutral axis limited web cracks to the crack width at the level of A_s. Such bars can also be counted as part of A_s to the extent consistent with their lower strains and shorter internal lever arm for flexure.

The author cannot make firm recommendations in this area. He feels that the problem may be real in a 30 in. deep member when Gr 60 bars are used for A_s. The French tests are the best guide for requirements at this time; but a beam 6 ft deep would probably need *more* face steel for the same effectiveness. A thickness of 3 ft would also raise questions about whether only face steel (for appearance) would not leave much wider internal cracks (from shear relaxation) which might influence durability under some exposures.

4.10 BEAMS WITH REINFORCING AT SEVERAL LEVELS

When reinforcing is placed in two layers, it is usually satisfactory to consider the f_y stress at the centroid of the steel. However, if the bars are in many layers or are distributed over all faces, as in a column section, some bars will be near the neutral axis and have a stress less than f_y. In such cases strains as well as stresses must be considered. This is not overly complex when strain triangles are used after the fashion already used for compression steel, as illustrated by Fig. 4.3 and the examples of Sec. 4.5 and Fig. 3.15.

Enclosure III

7.18.3 Design Provisions

The ACI code[104] now requires

$$\rho \geq 200/f_y \tag{7.21}$$

where ρ is the minimum ratio of steel required for this purpose and f_y is the minimal yield stress of this steel. It is specified that this reinforcement be spaced not more than 46 cm (18 in.) on centers.

Equation (7.21) ignores most of the significant variables. A more ambitious code requirement is[105]

$$a_s \geq \frac{450x}{(x + 100)f_y} \tag{7.22}$$

for structural elements not directly exposed to atmospheric weathering and twice this much for elements directly exposed thereto. Here a_s is the required area of steel per unit width, in cm²/cm, f_y is in kg/cm², and x is the structural element's smallest dimension in a direction perpendicular to the steel in question. No reinforcement is required in directions in which the structural element measures less than 150 cm (5 ft). Equation (7.22) can also be put in the form

$$\rho = \frac{450}{(100 + x)f_y} \quad \text{where } x \text{ is in cm and } f_y \text{ is in kg/cm}^2$$

$$\simeq \frac{6.4}{(40 + x)f_y} \quad \text{where } x \text{ is in in. and } f_y \text{ is in ksi}$$

where ρ is the required ratio of steel. This provision does not take into account the characteristics of the concrete mix that is to be used, save that it calls for greater ratio of reinforcement in special concretes that tend to shrink more than mixes made with aggregates of normal weight. Even so, it is already too complicated for most practical applications. It may be preferable to compute the maximum ratios required, which correspond to, say, $x = 10$ cm (4 in.), if this is the thinnest slab or shell in the project under consideration. Then Eq. (7.22) can be put in the form of Eq. (7.21), to give $\rho = 128/f_y$, with f_y in psi, for unexposed structural elements, and twice this much for elements that are exposed. In thicker elements, the required ratio decreases while a_s increases but does not exceed the equivalent of No. 4 bars of A34 steel 27 cm (10.5 in.) on centers or No. 6 bars 30 cm (12 in.) on centers, depending on whether the concrete surface is or is not exposed.

Vertical cracks having a maximum width at midheight of deep beams are classical in members whose lateral faces are insufficiently reinforced with longitudinal bars (Fig. 7.16).[106] Criteria such as Eqs. (7.21) or (7.22) coupled with a limitation on the maximum spacing of steel for volumetric changes lead to designs that are usually satisfactory from this point of view.

"Reinforced Concrete Engineering" Volume 1
Edited by Boris Bresler, John Wiley & Sons, Inc.
1974